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GROUTING FOR GROUND WATER CONTROL IN UNDERGROUND MINING

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ABSTRACT

This paper is a general review of grouting techniques as applied at the various phases in the construction, development and operation of an underground mine.

The assessment of the groundwater problem and the general approach to grouting design and procedures are described, together with a number of case histories, from both the United Kingdom and overseas, which serve to illustrate the different forms of grouting.

INTRODUCTION

There are three main phases in the "life" of an underground mine where some form of grouting for the control of groundwater may be required:

(i) Development and Commissioning - during the construction of shafts, surface drifts and preliminary underground developments to gain access to the orebody.

(ii) Production/Operations - dealing with groundwater problems at the production face and when driving additional underground roadways, shafts, etc.

(iii) "Shut-down" - initial sealing of shafts and roadways and continuing remedial work as necessary.

Within these general areas different forms of grouting will be required to solve the various groundwater problems that are encountered. These

may range from the permeation grouting of waterbearing alluvial and granular rock formations to "void-filling" grouting operations such as the backwall grouting of shaft and tunnel linings and the formation of underground plugs and dams.

Grouting at the development stage of a mine is usually a pre-planned operation based on detailed borehole investigations, and is aimed at reducing the difficulties for shaft sinking or tunnel driving. Groundwater problems, in association with phase (ii) above, and to a lesser degree phase (iii), are generally the results of unexpected inrushes or increasing seepages and here the use of grouting is more a "firefighting" application. In most instances the mine development and mining method will be designed to minimise the risk of such water inflows during production.

In the following sections of this paper the various groundwater problems and methods of grout treatment are addressed in some detail. The different types of grout employed and the approach to design of typical grouting programmes are described. Finally the various methods are illustrated by reference to examples of grouting operations carried out in recent years by the authors company.

ASSESSMENT OF GROUNDWATER PROBLEM AND METHODS FOR GROUTING

Purpose of Grouting

Although grouting has a number of quite different applications the main purpose, in the context of this paper, is always to eliminate or reduce the flow of groundwater into an existing or proposed underground excavation. Grouting is only one of several methods of ground treatment for excluding water which have to be assessed on their respective merits for each situation. However, grouting does provide the benefits of a permanent, or at least semi-permanent, ground treatment and the bonus of increased stability in some situations, as compared with purely temporary expedients such as dewatering and ground freezing.

The acceptable level of groundwater in any mine situation will depend on a number of factors including the type of mine, particular client requirements, safety aspects, pumping economics, and environmental considerations. In some instances the nature of the mineral involved dictates that a "dry" mine is required. In other circumstances where perhaps pumping and disposal costs are low, mining can continue with little or no ground treatment and relatively high levels of water inflow.

The philosophy of British Coal (formerly the National Coal Board) with regard to controlling water is one of exclusion rather than pumping (Dunn 1982). In terms of shaft sinking and tunnelling it is considered that the present mining construction techniques can tolerate a flow of up to some 3.8l/s (50 gpm) into the excavation before action must be taken to reduce the flow. In the recent Selby mine developments, involving both shafts and surface drifts, the requirement was for a final inflow after permanent lining of less than one gpm. This can be compared with potential inflows from the major aquifer zones of several thousand gallons per minute, and gives an indication of the overall efficiency required from the grouting process. For instance a 99 per cent effective grout treatment is required in order to obtain a ten fold reduction in water make to the excavated shaft. This degree of cover is difficult to

achieve and explains the long time that is often required for satisfactory chemical grout treatments.

Origins of Groundwater Problems

Mine water inflows have a variety of origins. Access shafts or surface drifts will often encounter substantially horizontal - lying aquifers before reaching the mining horizon. These aquifers may be relatively shallow water-table aquifers or deeper confined aquifers, and their nature may range from high intrinsic permeability alluvial deposits or sandstones to intensely fractured and vuggy limestones. Underground tunnels, drivages, and galleries, may encounter major faults which can transmit groundwater through relatively impermeable zones from such aquifers into mine workings. Similarly the presence of faults and other discontinuities in the rock mass may present water problems for developments under bodies of water such as lakes, reservoirs, large rivers, and the sea, or to developments in close proximity to abandoned mines, unsealed or poorly sealed shafts and boreholes which are likely to be flooded. (e.g. Dunn 1982, Wilson 1985, Garritty 1983, Slatcher 1985).

In most respects all of these situations represent water inflow problems related to naturally occurring primary and secondary permeability in the ground. To these must be added the range of problems connected with induced or modified permeability due to mine development and mineral extraction. Channels may be created through basically impermeable roof strata to connect with the sources of groundwater described previously. (e.g. Singh 1982, Massey 1984).

It is essential therefore that the nature of the groundwater problem is thoroughly investigated and fully understood to enable the optimum method of ground treatment to be selected and, in the case of grouting, for the appropriate form of grouting to be introduced. (Daw 1986).

Grouting Methods and Materials

The various grouting methods can be described by reference to the mechanisms by which the groundwater flows are eliminated or reduced.

Permeation Grouting

In permeation grouting the grout material penetrates the interconnected porous structure of the soil or rock which may comprise both the intergranular voids and the fissure network. Whilst in most instances the fissure permeability represents the major contribution to the total permeability of the ground and, hence, the main agent for transmitting groundwater flow to the excavation, there are instances where intergranular permeability is equally important. This will be most evident in the shallower coarse sand and gravel aquifers but can also occur at depth in medium to coarse grained sandstones, e.g. in U.K. the Bunter Sandstone, Basal Permian Sands and Coal Measure Sandstones.

In such aquifers it will be necessary to inject chemical grouts in order to achieve the required penetration of the intergranular network and also the finer fissures. The choice of a particular chemical grout will depend on a number of factors including permeability and pore size of the aquifer rock, cost, strength, and permanence requirements of the grout, and environmental considerations. In order to penetrate the "finest"

rock structures that still pose a groundwater problem, it may be necessary to go to the lengths of "clarifying" the chemical grout component solutions by filtration and/or centrifugation.

For wider fissures, and a width of about 2 x 10^{-4}m is usually considered to be the lower limit, standard cement-based grouts will be used. Since these wider passages will generally be carrying the major water flows it is the usual practice to inject the cheaper cement grout as the first phase. In some instances this procedure will reduce the inflow to a tolerable level, whereas in other cases a secondary treatment using chemical grout will be necessary.

Hydrofracture Grouting

Although both intergranular and fissure grouting is described here under the heading of permeation grouting, the actual penetration process is obviously quite different in a fissure than in a pore. However, in both instances the injection is carried out at pressures insufficient to disturb the ground structure. The grout advances steadily displacing air and water outwards with the predominant direction of flow being that offering the least resistance, i.e. the path of highest permeability. In some circumstances, usually in relatively shallow alluvials, it is permissible to use hydrofracture grouting where deliberate overpressuring is used to either widen existing fissures or create new fissures. This procedure has the advantage of rapidly creating direct access through low permeability ground to a more permeable and treatable zone from the widely spaced array of injection holes. In addition, by creating new passages, a greater injection "surface" is available for the grout to impregnate the ground. From relatively shallow surface boreholes the "tube-a-manchette" injection technique would most likely be used, and both chemical and cement grouts could be employed depending on the particular application.

Squeeze Grouting

Problems sometimes occur where unconsolidated, but relatively impervious deposits under high groundwater pressure and within an otherwise competent rock structure, need to be consolidated. These may comprise finely fractured or pulverised rocks, silts, soils, millonites, mud runs, etc., none of which can be permeated with grouts. The squeeze grouting technique is used by which grouts are used to apply high pressures to the ground to squeeze out excess pore water and consolidate the unstable material by increasing its density and shear strength. Consolidation is achieved by either forming a grout "bulb" which does not penetrate the soil or preferably by deliberate hydrofracture using a grout of limited capability to penetrate far. The use of hydrofracture enables a larger zone of ground to be stressed from a single grout pipe. Viscous fluid grouts are required but a number of combinations and variations of approach are possible, ranging from neat cement grout to clay cement grouts and thickened chemical grouts. (Greenwood 1982).

Void-Filling Grouting

In void-filling grouting the requirements and methods will usually be quite different from those described above. In most instances cementitious grouts will be employed, often with fillers or cement replacement materials in order to reduce costs where large volumes are

involved. Frequently chemical admixtures will be incorporated in order to impart the optimum combination of fluidity, workability, and setting characteristics.

In back-wall grouting, the first stage of the operation will be to fill the bulk of the cavity between the shaft or tunnel lining and the excavated profile. The choice of grout will be dictated by the anticipated volume of the void; as the void volume increases the fraction of cement replacement material used in the grout will generally be increased. Despite the use of admixtures to control the settlement of particles that occur in a cementitious grout prior to setting, some "bleed" often occurs. This can result in bleed channels through which water can still migrate around the lining. The second stage of backwall grouting serves the dual purpose of sealing these bleed channels and also of locking the lining into the surrounding rock, and is effected by the injection of thin neat cement grout mixes.

After the construction of a shaft plug or tunnel dam, thin cement grout mixes are used to lock the structure, and on occasion to form a cut-off curtain by injecting an array of holes drilled out from the plug or dam into the surrounding strata.

Combined Techniques

In some circumstances grouting has been used in combination with other methods of groundwater control during shaft sinking and tunnelling. In particular, the incorporation of groundwater pressure relief wells, together with grout injection, was used at two of the Selby shaft sites to facilitate excavation through deep sandstone aquifers (Fotheringham 1983). The two specific applications were: (i) where weak rock near the face of the excavation would have been subject to collapse under the action of the ground water pressure, and (ii) where the shaft lining would have been subjected to excessive hydrostatic loading during backwall grouting. Such a combination of grouting and depressurising can be expected to be very effective, even if the grout cover is only 80 per cent complete.

Depending on the circumstances, the relief-wells may be sunk from surface and the water pumped by submersible pump (Juvkam-Wold 1982), or may be sunk as a steep conical array from a temporary shaft sump above the aquifer. (Fotheringham 1983, Scott 1983).

Although ground freezing and grouting are not generally considered as suitable combination techniques, they were used for successive sections of waterbearing strata at the Gascoigne Wood Drift site, Selby Mine and the somewhat novel techniques used at the "overlap" zone are of interest. (Daw 1983).

GROUTING AT DEVELOPMENT STAGE

Introduction

In most mining situations where groundwater presents a problem and grouting is employed it is at the development stage that the major grout treatments take place - the purpose being to enable the shafts or drifts to be constructed both safely and efficiently. Four particular applications of grouting can be considered - pre-grouting from surface

boreholes, cover grouting from the shaft sump or tunnel face, backwall grouting, and in certain circumstances the placement of shaft plugs and drift or roadway dams.

Pre-grouting from Surface

The use of the pre-grouting method from surface boreholes has, in the U.K. and North America at least, been restricted in general to relatively shallow depths of up to about 200 to 300 metres. (Jones 1979). In other parts of the world, e.g. South Africa, U.S.S.R., where particular hydrogeological conditions exist, the method has been used at considerably greater depths. (Dietz 1982, Kipko 1984).

Probably the main application has been when a relatively "thin" alluvial or fractured rock aquifer zone occurs at shallow depth and other processes such as dewatering, ground freezing, or cover grouting, are considered impractical or uneconomical. The main advantage is that the grouting is carried out prior to excavation and hence does not interrupt the sinking process. In addition, it is not subject to the space and environmental problems encountered when grouting in a shaft sump. However, the two methods are not usually competitive, being dependent on specific ground conditions, and it is possible for both approaches to be used in the same shaft or drift.

In the case of a shaft a ring of vertical grout holes, typically six to twelve in number, are drilled around, and somewhat outside the periphery of the shaft. (Fig.1). The actual number of holes will depend on the diameter of the shaft and the results of the ground investigations. Often, and particularly for large diameter shafts, a central hole is also drilled, which may be used initially as a test hole.

In the case of surface drifts, rows of grout holes will be drilled along the line of the drift, with the number and spacing of holes chosen to give adequate coverage of that area of the drift which will intersect the aquifer zone. (Fig.2).

In rock grouting the drill holes will most often be drilled to full depth, with the grout injections carried out in ascending stages using borehole packers. Other techniques, such as the "tube-a-manchette" method may be employed for shallower alluvial aquifers. A sequence of primary and secondary treatment will normally be adopted.

Cementitious grouts are most commonly associated with pregrouting although where conditions have dictated a special requirement chemical grouts have also been used. In such cases a larger number of holes may well be required in order to obtain the required grout penetration for closure of the "curtain".

Cover Grouting

Probably a more common approach, and certainly that used in most U.K. coal mine developments, is to grout from within the excavation and ahead of the advancing shaft sump or tunnel face. (Keeble 1981, Pocock 1982, Black 1982). This has the advantage of being a "closer" controlled grouting process than pre-grouting from surface boreholes, as the shaft sump can be taken quite close to the aquifer zone. In addition, the hole drilling requirements are not so stringent, as the hole lengths are

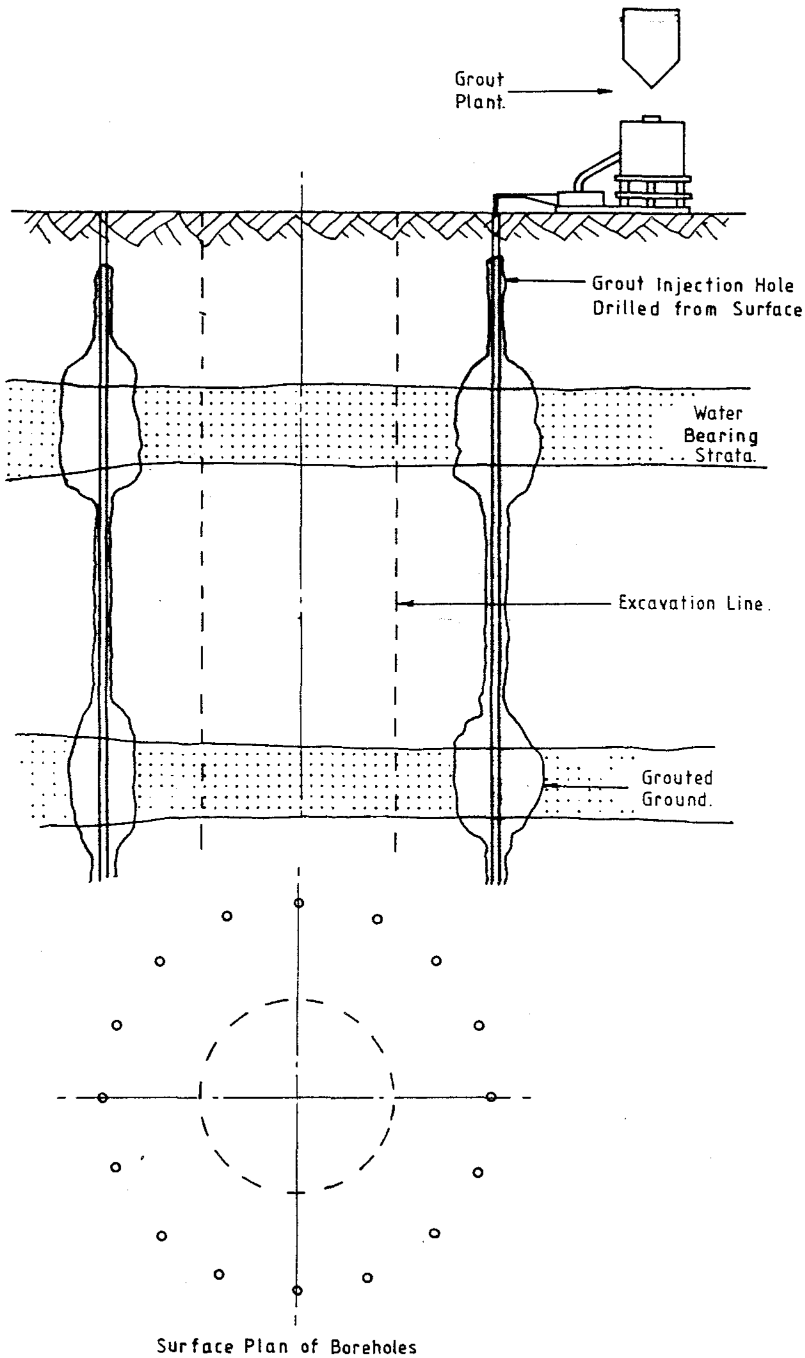


Figure 1 : - Typical Arrangement of Pregrouting from Surface - Shafts.

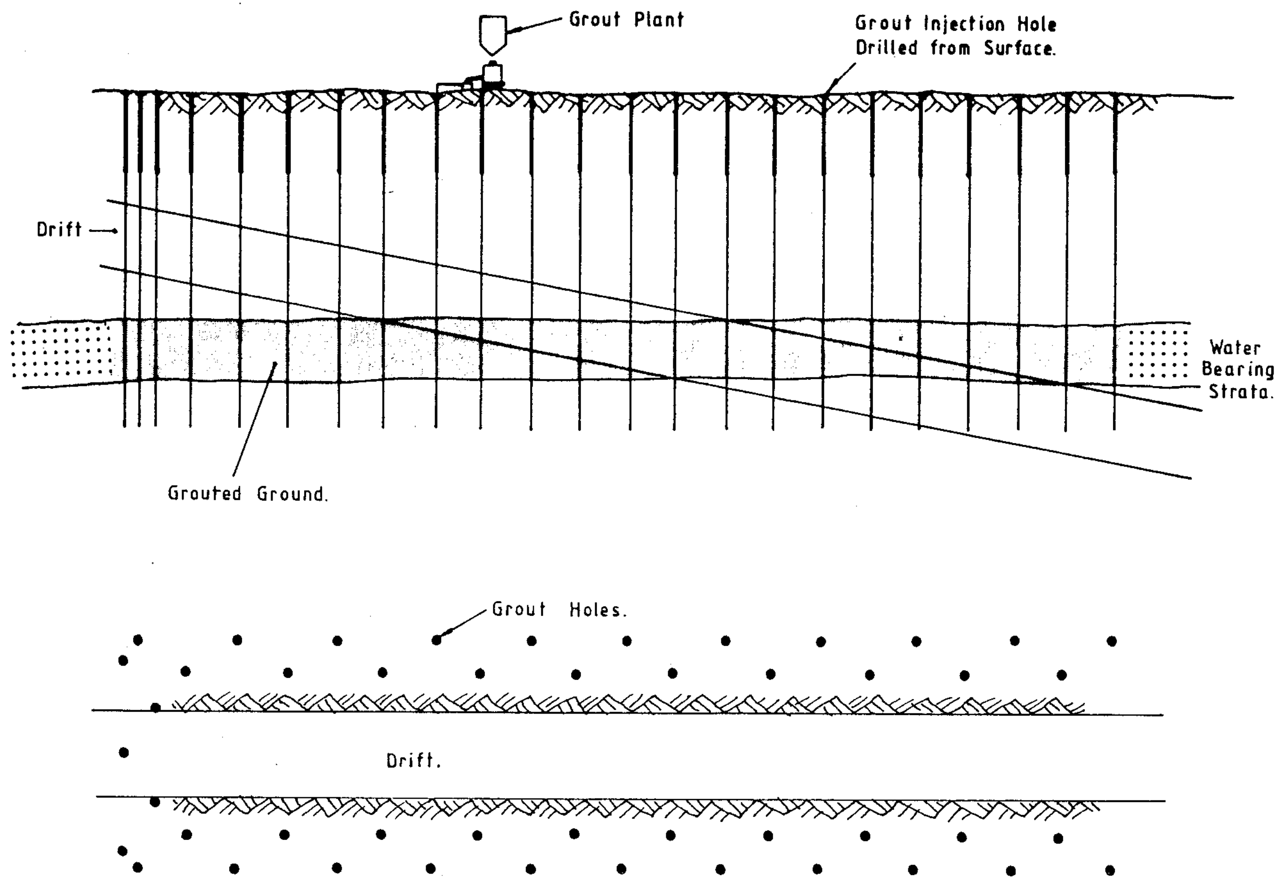


Figure 2 :- Typical Arrangement for Pregrouting from Surface - Drifts & Tunnels.

generally much shorter. However there are a number of disadvantages as mentioned previously.

On the basis of a detailed pre-sinking ground assessment and regular probe hole investigations ahead of the face, the excavation will be taken to within some 5-10m of the aquifer zone requiring the grout treatment. A "cone" of injection holes are then drilled through valved standpipes sealed into the shaft sump, and fanned out from the proposed excavation line of the shaft. A typical grout cover may be about 30m in length, and if an extensive aquifer is intersected, two or more overlapping covers may be required in order to achieve a full grout treatment (Fig.3). Within each cover the holes will be drilled and injected in depth stages of perhaps 3m, or in some instances extended until a certain level of "watermake" is encountered before grout is injected. In rock where the main waterflow channels may be vertical and sub-vertical fissures, it can be beneficial to "spin" the drill holes (Fig.4) in order to give a better chance of intersecting all the fissures.

A central test hole will often be used to monitor the effectiveness of the grout treatment in restricting potential water inflows to the shaft.

All grouting plant, including pumps and mixing tanks are located within the shaft, and usually in the actual sump, although sometimes use is made of the shaft sinking stage. Both cement and chemical grouting can be carried out in this manner. In circumstances where very highly penetrating chemical grout is required, a special grout clarification plant will be set up on surface close to the shaft, and the refined components are then taken into the shaft in special batching tanks.

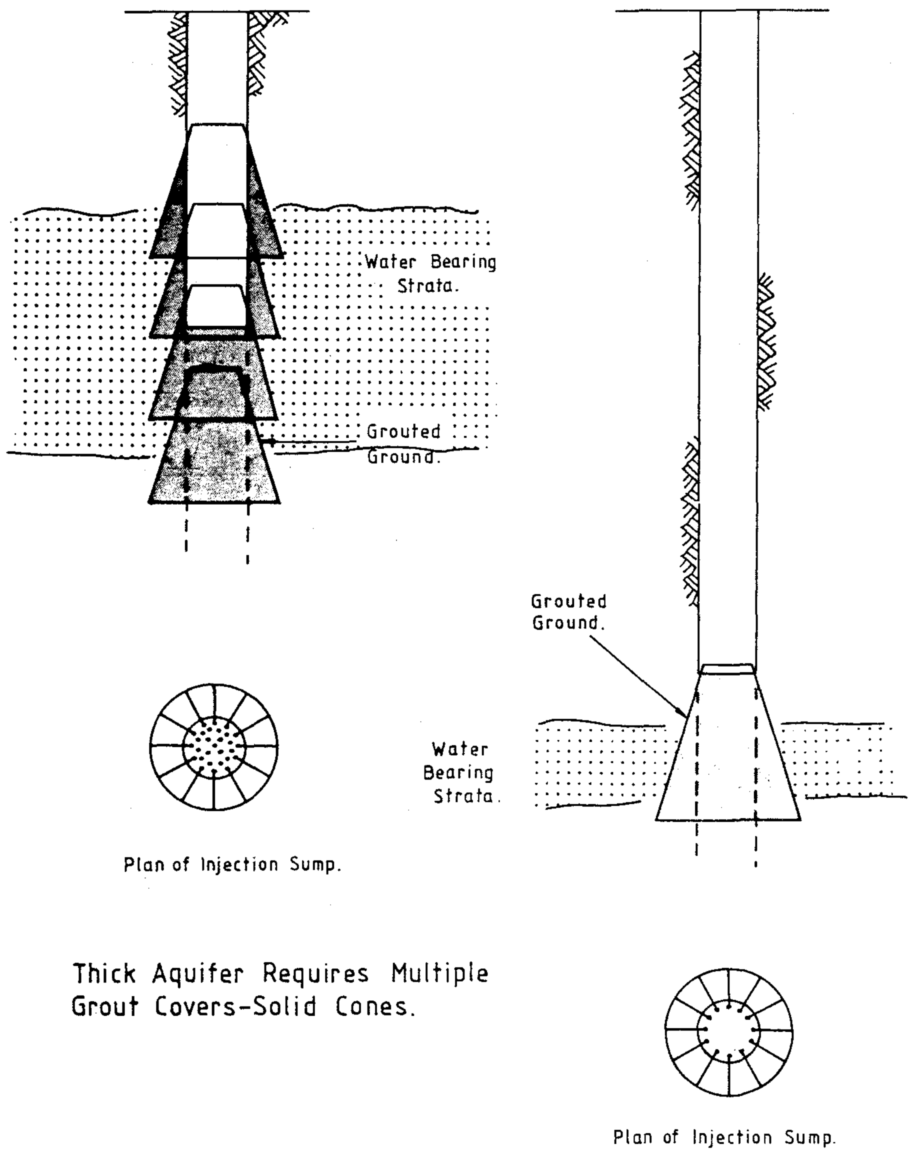
A similar approach is used for grouting from the face of a drift (Fig.5) although various modifications have to be made to the grout mixing "set-up" depending on the differing geometry of the drift and the space available at a particular face.

Backwall Grouting

Backwall grouting is the process of filling of the void between the excavated rock face and the installed shaft or tunnel lining. Efficient backwall injection and drying off any residual water seepages behind the lining can be as important as any ground injections to the successful completion of the shaft or tunnel.

A typical procedure for backwall grouting of a section of shaft is shown in (Fig.6). Grouting generally commences after the concrete in a particular length of lining has been cured adequately, with injections proceeding upwards from the lowest ring through injection pipes cast into the lining. Injections are carried out in different phases using progressively increasing grout pressures until all leakages are eliminated over the particular length of lining.

In general, three rings of holes are drilled out and grout valves fitted to each injection pipe. Grouting commences with thin cement grout (W:C 10:1) until connections between the holes are established, then the grout consistency is thickened and the holes closed off. Injection continues with a thin mix and the grout is allowed to rise and establish connections with the next ring of holes. The grout is then thickened and the holes closed off as before. The injection point is moved to this



Thick Aquifer Requires Multiple Grout Covers-Solid Cones.

Thin Aquifer Requires Single Grout Cover-Hollow Cone.

Figure 3 :- Typical Arrangements for Cover Grouting - Shafts.

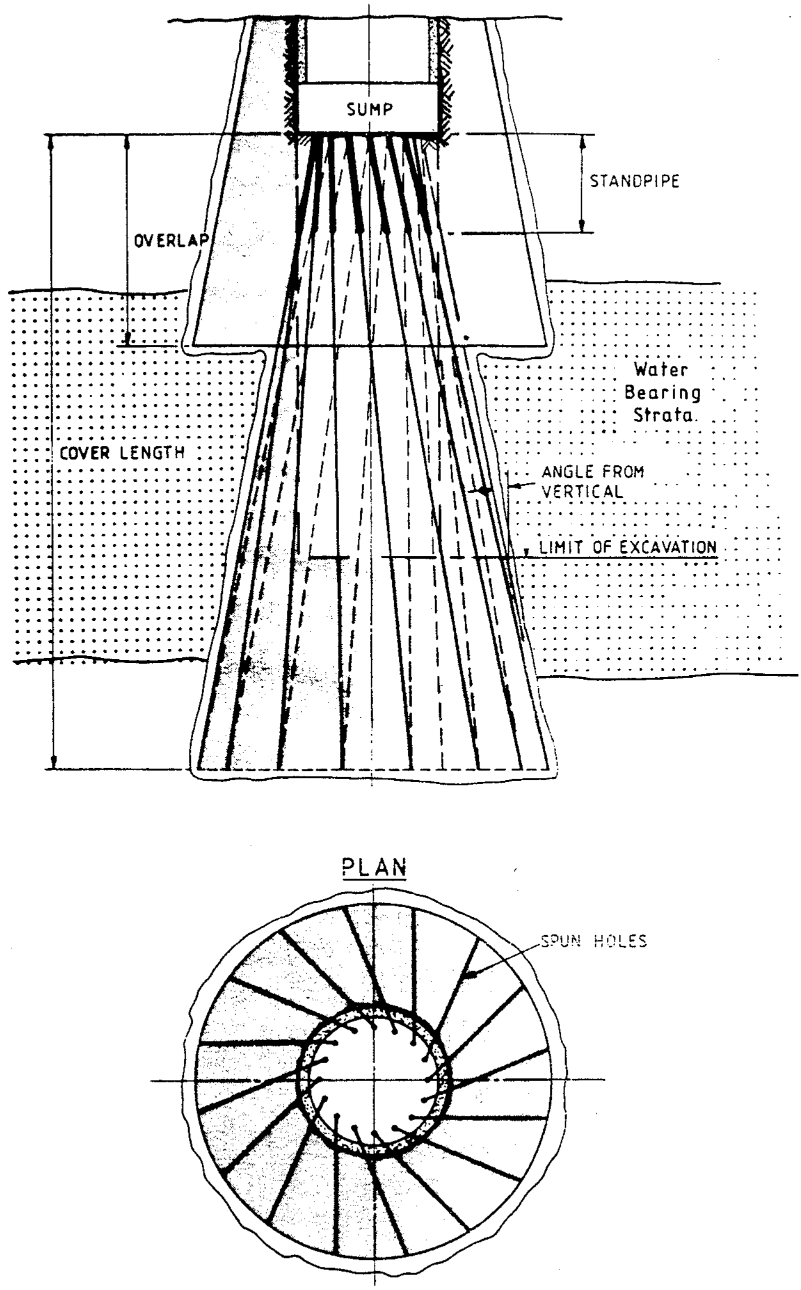


Figure 4 :- Illustration of "Spinning" of Drill Holes for Cover Grouting.

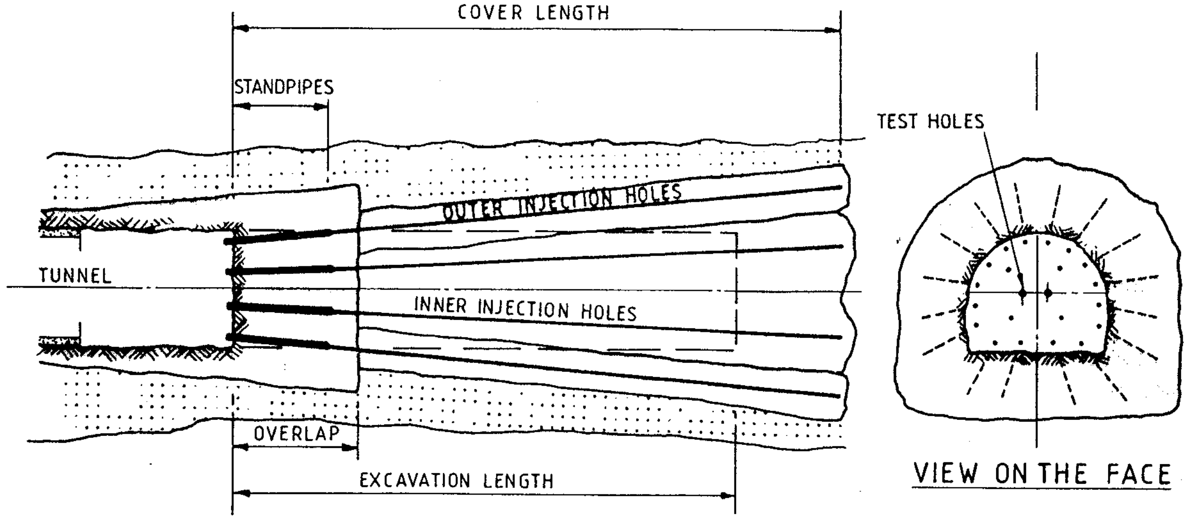


Figure 5 :- Typical Arrangement for Cover Grouting - Drifts & Tunnels.

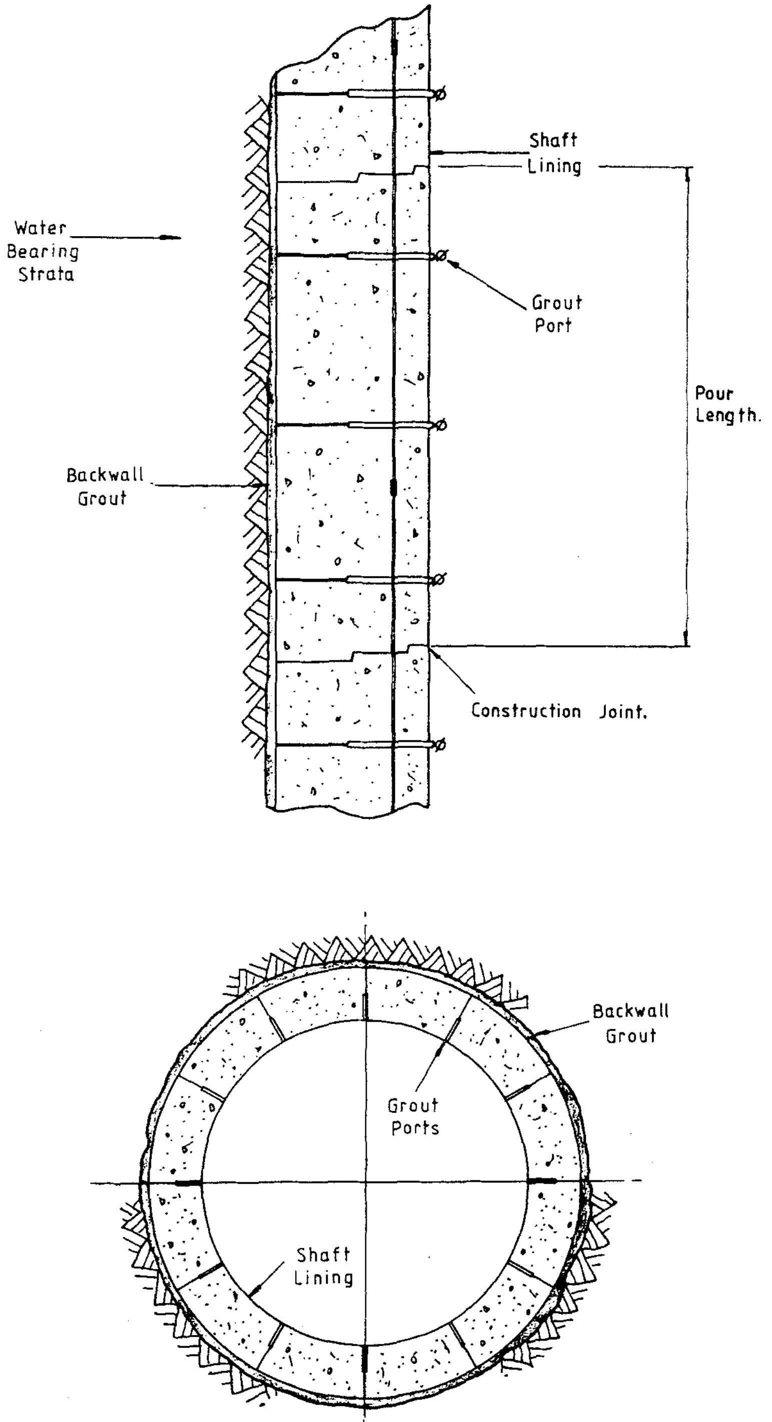


Figure 6 :- Typical Arrangement for Backwall Injection in a Shaft.

second ring of holes and the procedure repeated. Any small leaks are dealt with by caulking with small wooden wedges or lead wool and temporarily increasing the grout consistency.

The grout is allowed to set, then the injection holes are drilled out and a second pass made with a thin cement grout mix. The injection is terminated at a pre-determined finishing pressure.

Cement grouts for backwall injection are best mixed in a high shear mixer and usually contain a plasticising admixture to give better dispersion of the cement particles. The thicker mixes used to fill voids will contain anti-bleed admixtures which, by increasing the viscosity of the aqueous phase, reduce the rate at which settlement of cement particles occurs.

Shaft Plugs

Despite detailed pre-investigation work and forward probing there are rare occasions where it is necessary to cast temporary plugs of concrete in the sump of the shaft prior to further progress with the excavation. Whilst plugs and dams will be discussed in more detail in later sections, there are two applications that are relevant to this development phase of the mine.

For example, in an extensive aquifer zone where more than one grout cover is necessary it may not be possible to find a suitable section of competent rock to establish as a sump for the second or subsequent grout covers. In these circumstances it can be beneficial to cast a "consolidation" plug in the sump, designed to provide the resistance required for satisfactory setting of standpipes and high pressure grouting. (Fig.7). Once the grouting is complete and the water zones are satisfactorily sealed, the plug is removed and shaft excavation continued.

The other application can be regarded as an "emergency" plug, required if the shaft is subject to a major unexpected inrush of water. In the worst situation this may cause a temporary abandonment of the shaft. A concrete plug may then have to be cast underwater before the necessary remedial action can be taken, the water pumped out and the shaft recovered.

A detailed account of the design of the various types of underground plugs is given by Auld (1983).

GROUTING AT PRODUCTION PHASE

In most underground mine developments, the need for grout treatment will not be anticipated. Either drivages and ore extractions will be in "dry" conditions due to the absence of aquifers or by deliberate design to avoid groundwater problems, or the level of inflow can be handled conveniently and economically by pumping.

However, on occasion unexpected groundwater inrushes are encountered for which some form of grout treatment proves beneficial, and indeed essential, before further mining progress can be made. The possible causes of the inflow are numerous, e.g. the intersection of unknown fault zones, or uncharted former shafts or workings which have become flooded. In the act of mining itself a strata zone of modified permeability is

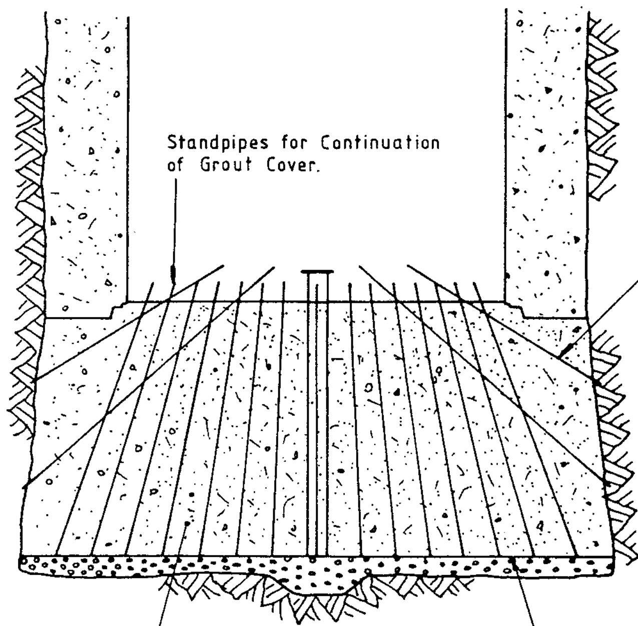


Figure 7 :- Arrangement for Consolidation Plug in Shaft Sump.

created around the excavation. The dimensions of this zone are dependent on several factors including seam thickness, length of the working face, its location relative to previous workings, and the rate of advance. The overlying beds in this modified zone exhibit broken, fractured, rock behaviour and sometimes react as separated beams. If this zone intersects or connects to an overlying aquifer or surface water body then water will flow into the working areas.

Where inflows are fairly localised and perhaps not too severe, and rock conditions are stable, a fairly simple remedial grout treatment with a limited number of injection holes will be carried out from the face. The pattern of holes will be designed to give the best chance of fully intersecting the inflow zone. Injection procedures will be as described previously.

In instances where inflows are greater and access to the inflow point is difficult, attempts may be made to grout from surface boreholes. However, such procedures would probably not be applicable at great depths and it would be essential to have identified the cause of the groundwater problem beforehand, since a large number of such boreholes would be unacceptable.

The use of underground plugs is probably more widespread in this phase of mining than in the development phase. In addition to the emergency and consolidation plugs mentioned previously, two other categories of underground plugs or dams may be employed during the production phase.

Precautionary plugs are normally constructed in underground roadways to limit the area of flooding should water inrushes occur. Watertight doors are built into them which can be shut when any danger of flooding arises. They are installed as a safety measure prior to development into areas expected to be water-bearing, and such plugs are designed to withstand full hydrostatic pressure from surface level. (Fig.8).

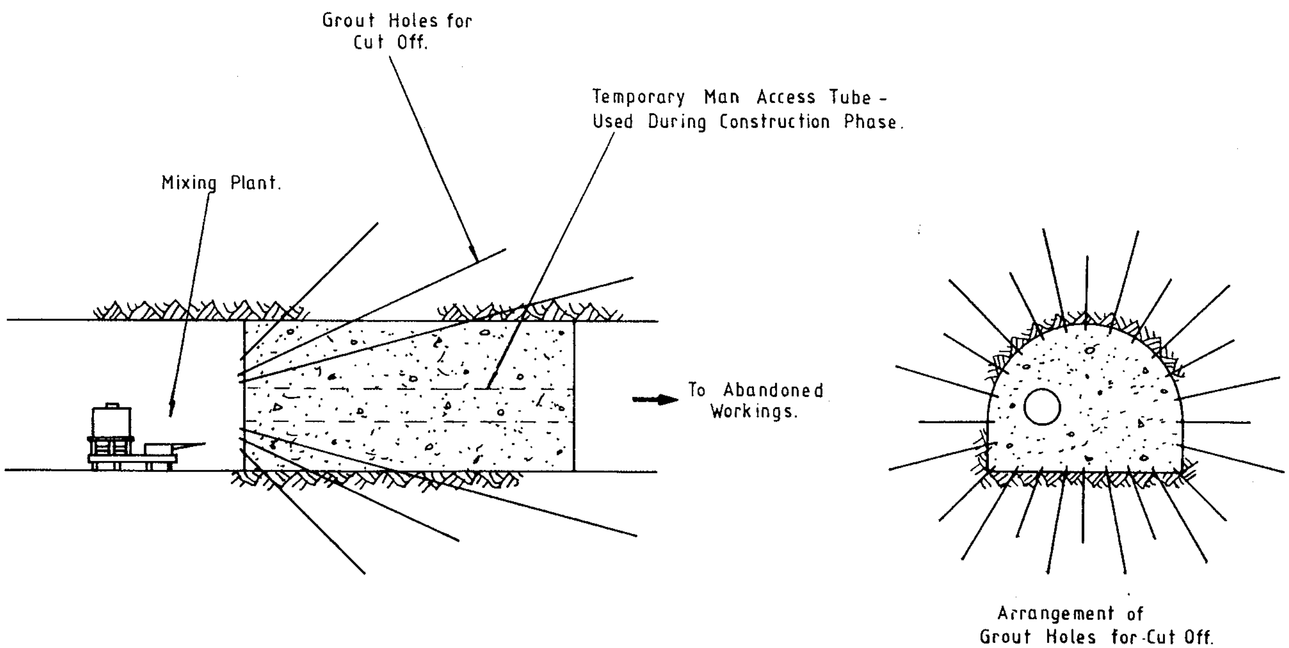
Control plugs are introduced to seal off or control the inflow of water from abandoned mining areas. Boundary plugs which are constructed in boundary pillars between adjacent mines to prevent migration of water from abandoned areas into current workings come into this category. No means of access to the sealed off areas is provided in control plugs, but drain pipes with valves are normally cast into them.

On occasion, some remedial grouting may be required to the access shafts and drifts during the period of the mining operation, to eliminate leakages, often minor, which have occurred through the linings.

GROUTING AT ABANDONMENT STAGE

As reserves are exhausted, and mines closed down, with pumps ceasing to operate, the groundwater levels recover and workings become water-logged. Such operations can put adjacent mines at risk and also new developments in those areas where the presence of old workings has not been well recorded.

It is very important therefore at this closure stage, in those mines where groundwater problems have existed that certain precautionary measures are taken to ensure that the workings are made as safe as possible from the hydrological point of view. This can inevitably



Typical Arrangement for Constructing Roadway Dams.

involve the various aspects of grouting.

Whilst there may be occasions where some limited injection grouting may be required to seal specific aquifer zones, by far the greater emphasis in this phase will be in large void filling grouting and in the casting of plugs and dams, as described previously.

The large void filling may involve complete backfilling of shafts or part plugging of specific zones within shafts and tunnels. Here large volumes of inexpensive grouting materials are required. Dams and plugs for sealing off particular areas of mines need to be designed with the required structural integrity to withstand maximum hydrostatic pressures that might develop.

APPROACH TO GROUTING DESIGN AND PROCEDURE

General

Whatever the form of grouting problem it is essential that the cause is investigated fully to enable proper grouting design to be implemented. The degree to which this assessment can be conducted will obviously be dictated by the particular situation. For example, the time available for a borehole investigation programme prior to shaft sinking will be quite different from that available when a sudden inrush is encountered underground. In the latter case a fairly instant solution is usually required and it is the actual experience of the operators 'on-the-spot' that is of most importance. When more time is available for preplanning, a detailed borehole investigation programme is carried out and this is best illustrated by the procedures typically adopted for a new shaft or drift project.

Site Investigation Methods

Prior to a new mine development, a large number of exploratory boreholes will have been drilled over the area, and the presence of any major aquifer zones overlying the orebody will have been identified. Once the positions of access shafts and/or surface drifts have been fixed, a series of hydrogeological test boreholes will be sunk. in the case of shafts these will ideally be on, or near, the proposed centrelines, and for drifts several boreholes may be required along the line of the drift, in order to intersect the aquifers at the appropriate depths.

The boreholes should, when possible, be fully cored with geological and geotechnical logging of the core material. A complete suite of geophysical logs will be run in the boreholes, together with in-situ hydrological testing of the identified aquifer zones. From the point of view of ground treatment design it is these hydrological tests that are of particular importance (Daw 1984).

Some on-site index testing of core material, together with more specific laboratory testing of core and groundwater samples is also necessary for the complete ground treatment design.

Hydrogeological Assessment

The results from the hydrological testing provide the location of the major aquifer zones, their average permeabilities and groundwater pressures, and enable estimates to be made of the potential water inflows to the proposed shaft or drift. If these levels warrant some form of ground treatment, then it is the correlation of the data from all sources that enables the most appropriate method to be selected. (Black 1982, Forrest 1979, Harris 1985). In very general terms ground freezing, being a very costly operation, would be employed where an extensive aquifer was met at relatively shallow depths, and unstable ground conditions were anticipated. Grouting might be considered more appropriate where "thinner" aquifers were encountered and more competent ground conditions (Forrest 1979). Dewatering and groundwater pressure relief wells would rarely be used as a sole method except for very shallow excavations such as for foreshafts and preliminary sections of drifts. However, in practice, a large number of interacting factors have to be taken into consideration and the demarcation between methods is never as simple as this.

If grouting is the selected method then it is necessary to examine the in-situ test data, the laboratory permeability data, the discontinuity logs of the core, and to study the core material itself. On this basis it is possible to determine the main cause of the permeability, i.e. fissures or intergranular porosity, and to make some assessment on the likely magnitude of the average pore diameters and fissure widths.

The most appropriate grout can then be selected, having also taken into account the groundwater analysis, local Water Authority requirements on disposal, availability of grout materials at the particular location and cost.

Grout Cover Design

The aquifer depth and thickness determines the number of grout covers that are required. The length of a single grout cover is restricted to about 35-40m, both by the limited accuracy with which the grout injection holes may be drilled by rotary percussive machines, and by the increased spacing between the injection holes as they fan out with increasing distance from the injection sump or face. If the zone can be treated with a single cover this takes the form of a hollow cone or curtain. If multiple covers are required, the grouted zone comprises a series of interlocking solid cones as shown in Fig.3.

The precise elevations from which each grout cover will be injected are selected after close examination of recovered core samples from above and below the aquifer as well as from the aquifer rock itself. Where possible, the injection sumps/faces will be chosen so that the grout standpipes can be installed in competent strata. The steel standpipes are usually sealed in place with cement grout and their length is governed by the rock strength such that the necessary grout injection pressures and rates can be achieved without causing damage to the shaft sump. In the differing aquifer conditions at various Selby Mine sites, standpipe lengths varied between 3m and 15m (typically 3 to 6m).

The actual geometry of the grout cover will depend on a number of factors which include the projected strength of the grouted rock, the groundwater

pressure, the method of excavation, the time delay before installation of the permanent lining, and the degree of water control required. Correlation of the selected grouts' properties with those of the aquifer rock enables the necessary grout hole spacing to be determined and hence the number of grout holes to satisfy the overall cover geometry. In turn, the required grout injection volumes per hole at the specific horizons can be estimated.

Grouting Procedure

The excavation face is advanced to the elevation from which the grout cover will be formed, and the injection standpipes are installed. A hole, typically 89mm diameter, is drilled to the required depth and in dry ground the hole is flushed out and thick cement grout tremmied in. The standpipe, typically a length of 2" nominal bore high pressure steel pipe, is then lowered into place. If, however, the hole is making water, a mechanical packer-type standpipe is used. It is placed in the hole and thick cement grout is injected through the standpipe. When a return of thick grout is obtained around the annulus, the packer is expanded to seal and the standpipe grouted in place under pressure.

Once installed, the standpipes are drilled out to 0.5m beyond their ends and pressure tested. If secure, the grouting works can then commence. The usual procedure is to advance the cover in a number of descending intermediate stages. The length of each stage is defined after close examination of both recovered core samples from the exploratory boreholes, and recorded water inflows from probe holes drilled into the zone to be grouted.

Each injection hole is drilled through a stuffing box, which is used to control any sudden large water inflows. Drilling is continued with the water inflows being checked and recorded every metre, until either an arbitrary maximum water inflow (say 50 gpm) or the stage depth has been reached. The hole is then injected with grout. Cement grouting will commence with very thin mixes which are made progressively thicker until the maximum injection pressure is reached. The rate at which the grout mix is thickened is governed more by the behaviour of individual holes rather than by a set regimen. However, chemical grouting proceeds with the injection of a pre-determined quantity of grout, unless the maximum injection pressure is attained prior to this quantity being injected. The holes are drilled out and injected in a strict sequence to avoid drilling a hole adjacent to one which is either currently being, or has just been, injected.

Upon completion of the grout cover, its efficiency is checked by test holes drilled inside the cover, and the reduction in transmissivity and hence potential shaft inflow assessed by in-situ pressure recovery tests.

Injection Pressures

In general, grouting operations which are carried out underground involve drilling and subsequent injection of grout into rock containing groundwater at high pressure. The injection pressure exceeds this back pressure by the net injection pressure, and this governs directly the grout injection rate for a given rock/grout combination. However, it is important when attempting to increase the injection rate by increasing

the injection pressure, that the injection pressure is not allowed to rise to the point at which hydrofracture can occur of either the stratum in which the standpipe is installed, or the aquifer being injected. This would possibly result in new leakage channels being opened up, and lead to uncontrolled groundwater flow back to the excavation injection face.

A number of factors must therefore be taken into consideration when determining the maximum injection pressure to be used during a particular grouting operation. These will include the mechanical properties of the aquifer and of the stratum in which the standpipes are installed; the geostatic head at the base of the standpipe; the hydrostatic head against which the grout is to be injected; and most important of all, previous experience.

Testing

As we have seen, the preliminary grouting design is based on information obtained from a surface borehole, and this may well be modified in the light of additional information supplied by probe holes drilled ahead of the advancing excavation face. These probe holes provide a final opportunity to search for undetected aquifers before they are intersected by the excavation. Rather than simply relying on the discharge rate of the probe hole as evidence of the strength of the aquifer ahead, which can be misleading, an in-situ test based on the pressure recovery principle has been developed. (Black 1982, Daw 1984).

The test equipment comprises an accurate means of pressure measurement, such as an electronic pressure transmitter with digital readout or a high precision bourdon tube test gauge, and a test valve additional to the probe hole standpipe valve. (Fig.9). The test valve is opened and water allowed to flow from the aquifer, producing a depletion in pressure.

The valve is then closed after a known "flow period" and a measurable volume of water has been collected, and the recovery pressure monitored with time for a period of at least three times the flow period. The subsequent data analysis is identical with the conventional pressure recovery test from surface boreholes. (Daw 1984).

The test is also used in test holes drilled after grouting to determine the efficacy of the grout treatment. Although the presence of the grout curtain modifies the aquifer characteristics close to the test hole, the basic data analysis does seem to provide realistic values for the permeabilities of the grouted and ungrouted zones and allows very approximate estimates to be made of the average thickness of the grout curtain.

CASE HISTORIES

Introduction

The following case history reports have been chosen to illustrate the various forms of grouting used for groundwater control in underground mining, as described in the previous sections.

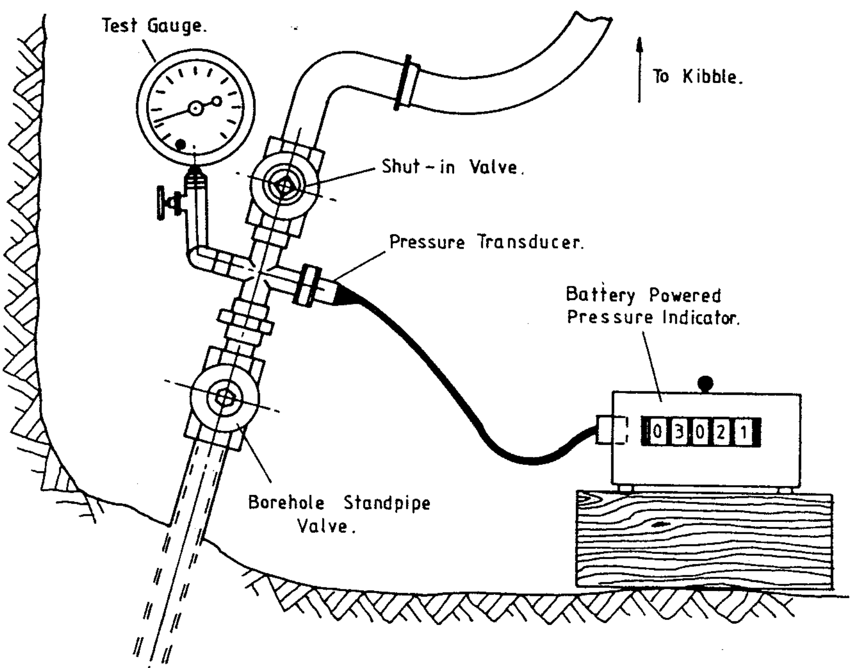


Figure 9 :- Pressure Recovery Test Apparatus for Use on Underground Probe Hole.

In most instances the reports cover fairly recent contracts carried out by Cementation Mining Ltd. in the U.K. but, where considered more appropriate, examples are taken from Cementation Company contracts overseas, specifically in North America and France.

Shaft Pregrouting - U.S.A. (Jones 1979)

Pregrouting from surface boreholes was used by The Cementation Company of America prior to the sinking of five mine shafts in the Coal Measures of the Eastern United States. In four of the shafts, in West Virginia, cement grouting was used whereas at the fifth shaft, in Alabama, chemical grouting was employed.

Cement Grouting

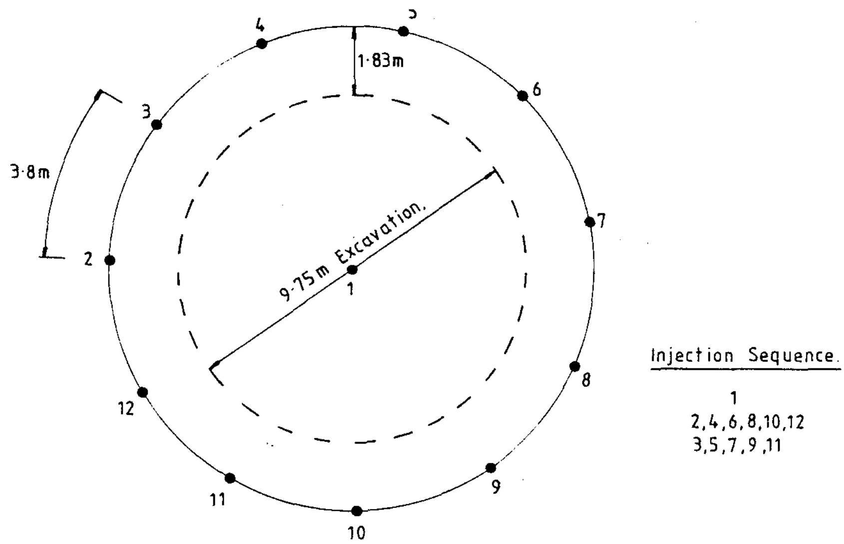
At the West Virginia site, packer tests had indicated maximum permeabilities of some 3×10^{-6} m/s over the 250 metre (approx.) proposed shaft depths, with the higher permeability zones related generally to fractured sandstones within the Coal Measures strata. The client elected for a cement grout pretreatment and this was carried out from 12 No. vertical injection holes at each shaft site. The arrangement of the injection holes is shown in Figure 10, with a central hole and eleven perimeter holes. The perimeter holes were drilled on a circle 1.83m outside the proposed excavation line for each shaft, which was either 9.75m or 8.53m diameter. The holes were drilled with NQ wireline equipment to full depth and then grouted in 12m stages from the bottom of the hole upwards. The outer holes were drilled and grouted in a primary and secondary sequence. A typical procedure was to drill a hole to depth, clean out, and then pressure test with a sodium silicate solution. This procedure helped to lubricate the fissures ahead of the cement injection. If the acceptance rate was greater than 0.1 l/s, cement was injected initially at a w/c ratio of 5:1 reducing in stages, with a 1500 l limit at each stage. Injection pressure limits were set at 11 kPa (1.60 psi) per metre of depth. Details of the grout injections are summarised in Table 1.

TABLE 1: DETAILS OF CEMENT PREGROUTING (Jones 1979)

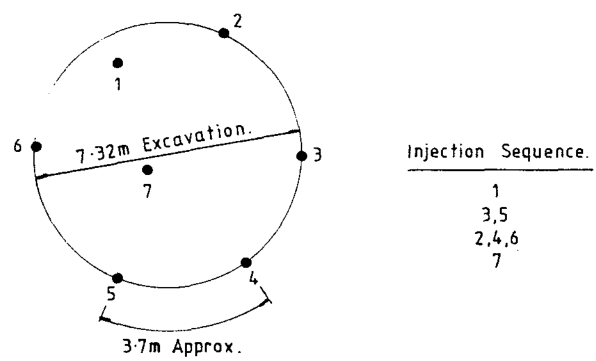
Shaft No.	Depth (m)	Tonnes Cement Injected	Approx. Volume Cement Grout l	Average Grouting Rate l/hr
1	247	69.6	73400	207
2	250	75.7	79900	278
3	236	58.5	61700	404
4	258	15.7	16600	200

Chemical Grouting

The 360m shaft in Alabama was in much more competent and lower permeability (less than 7×10^{-8} m/s) strata. Although the estimated water inflows were no more than 0.4 l/s from any zone, since the shaft was to be excavated by an experimental blind shaft boring machine, it was



(a) Cement Pre-grouting.



(b) Chemical Pre-grouting.

Figure 10 :- Arrangement of Grout Holes for Shaft Pre-grouting. (14)

considered essential to minimise all potential water inflows. Due to the small fissure widths, chemical grouting was necessary.

A seven hole injection pattern was employed, as shown in Figure 10, with the perimeter holes on a 7.32m diameter circle on the line of the proposed excavation. A previously drilled hole slightly inside this diameter, was incorporated and the "central" hole was offset to avoid the line of an earlier borehole which had been grouted off. The holes were drilled with BQ wireline equipment, and as with the cement injections, the drilling and grouting was carried out in a primary and secondary sequence. However, in this case, stage grouting was adopted with injections carried out below a mechanical packer, whenever drilling reached a permeable zone.

A modified silicate grout, Cemex D, was used with gel times adjustable between 15 minutes and 3 hours. The grout was mixed in special high speed shear mixers and injected by an hydraulically powered progressive cavity-type Mono pump. Pressure of injection was limited to 17 kPa (2.47 psi) per metre of depth. The details of the drilling and grouting operation are summarised in Table 2.

TABLE 2: DETAILS OF CHEMICAL PREGROUTING (Jones 1979)

Depth	Quantity of Grout Injected - litres							
(m)	1	3	5	2	4	6	7	TOTAL
0-30	0	17784	2453	1533	1226	0	0	22996
30-61	2453	0	0	0	0	0	0	2453
152-213	8279	3679	3986	0	1226	2760	0	19930
305-351	-	2453	3986	3373	6439	0	0	16251
Backfill	-	-	-	-	-	-	920	920
TOTALS	10732	23916	10425	4906	8891	2760	920	62550

Average Grouting Rate = 229 l/hour.

The paper by Jones (1979) gives a very interesting comparison of these two pregrouting operations in terms of drilling and grouting rates and costs. He concludes that in certain situations pregrouting with chemical grout can be more economic and effective than with cement grout. In all cases, however, it is essential that a proper ground assessment is carried out prior to grouting.

Cover Grouting for Shafts and Drifts - Selby, U.K.

Introduction

During the development phase of British Coal's new Selby Mine, cover grouting has been used in a number of different applications. The main aquifers intersected by the various shafts and surface drifts were the Bunter Sandstone, the Lower Magnesian Limestone, the Basal Permian Sands, and several thick Coal Measures sandstones. These aquifers occurred variously at the different sites between surface and depths of some 750m. Whilst the shallower and extensive Bunter Sandstone was frozen at all the shaft sites, the other aquifers, with the exception of the Basal

Sands at Gascoigne Wood, were cover grouted. Details of the grouting for the four sites where Cementation Mining Ltd. were involved are summarised in Table 3.

More details of these various grouting operations have been described in earlier papers for example by Keeble (1981), Black (1982) and Cockett (1984). As an illustration for the context of this paper we examine the grouting of the Lower Magnesian Limestone at Gascoigne Wood No.1 drift.

Drift - Gascoigne Wood No.1, Selby

At the Gascoigne Wood drift site the exploratory borehole programme had indicated a fractured and vuggy Lower Magnesian Limestone aquifer, with a potential inflow in excess of 380 l/s (5000 gpm) between depths of about 125 and 150m, corresponding to drift chainages of 540 to 650m approximately. This aquifer directly overlaid the highly permeable and relatively weak Basal Permian Sands, which were frozen in order to achieve the required stabilisation and impermeability required for drift excavation. It was decided that the limestone would be treated by a series of overlapping grout covers, each 30m long with a 12m advance between faces. The final grout cover would "interlock" with the frozen ground, with special construction procedures required in this area.

In practice, ground water was encountered at a higher level than anticipated and the grouting sequence was commenced from a chainage of 473m, and a total of 14 grout covers were then required to reach the frozen zone. This is shown schematically in Figure 11. The grout hole configurations were changed during progress of the passage through the aquifer zone. In the first six covers an 18 hole pattern was used, together with 4 central test holes. Alternate covers were drilled in either A or B patterns where the difference was in the location of central grout and test holes with patterns rotated to give a fuller grouting coverage of the central zone. In covers 7 to 12 the number of injection holes was increased to 20, with the location of the outer holes also rotated in alternate covers, to give patterns C and D. In the final two covers, for the critical zone up to the ice wall, a total of 39 holes were drilled in each cover. These various configurations are also shown in Figure 11.

In each cover 3m long standpipes were used and the holes were then extended in regular stages and grouted with a cement grout, although if an exceptionally heavy water inflow was encountered the stage length was reduced accordingly. By its nature the limestone proved very variable, and whereas in some covers inflows of several hundred gallons per minute were encountered in a single injection hole, in other cases the holes were completely dry. Details of all 14 grout covers are summarised in Table 4.

TABLE 3: SUMMARY OF COVER GROUTING AT FOUR SELBY MINE SITES

Site	Aquifer	Shaft Drift No.	Depths Range m	Potential Inflows l/s	No. of Covers (Holes)	Grouts	Quantity Injected	Other Details
Gascoigne Wood	Lower Magnesian Limestone	1	100-155	380 +	14	Cement	300 t	
		2	105-160	380 +	13	Cement	310 t	
						Cemex A2	4200 l	
Wistow	Lower Magnesian Limestone	2	240-255	76	1(48)	Cement	218 t	
						Cemex A2		
	Basal Permian Sands	2	259.2-263.1	2.6	1(16)	Cement	17.5 t	12 pressure relief wells
						Cemex A2		
Riccall	Lower Magnesian Limestone	1	417-426	7.6	1(32)	Cement	4.5 t	
						Cemex A2	70.90 l	
		2	423-429	9.1	1(32)	Cement	22.2 t	
						Cemex A2		
	Basal Permian Sands	1	426.9-432.5	9.1	1(144)	Cement	109.4 t	12 pressure relief wells and steel tubbing
						High Solids silicate	162750 l	
						Cemex A2	381900 l	
		2	429.6-433.6	5.6	1(96)	High Solids silicate	2410 l	
						Cemex A2	102300 l	
North Selby	Upper Magnesian Limestone	1	288.3-318.0	3.5	1(20)	Cement	24.5 t	H_2S gas present
						Cemex A2	74900 l	
		2	-	-	-	-	-	Dry, no gas present
	Lower Magnesian Limestone/ Brierly Rock	1	461.5-507.5	7.6	2 64	Cement	64 t	
					16	Cemex A2	365400 l	
		2	472.5-537	14	2 64	Cement	78 t	
					16	Cemex A2	32240 l	
	Ackworth Rock	1	526.4-575.0	88	2 72	Cement	205 t	2nd cover injected from sump plug
					48	Cemex A2	1597800 l	
		2	525-585	57	1(64)	Cement	170 t	2nd cover injected from sump plug
						Cemex A2	1701600 l	
	Shafton Sandstone	1	585-625	3.5	-	-	-	No ground treatment required
		2	594-631	3.8	-	-	-	No ground treatment required

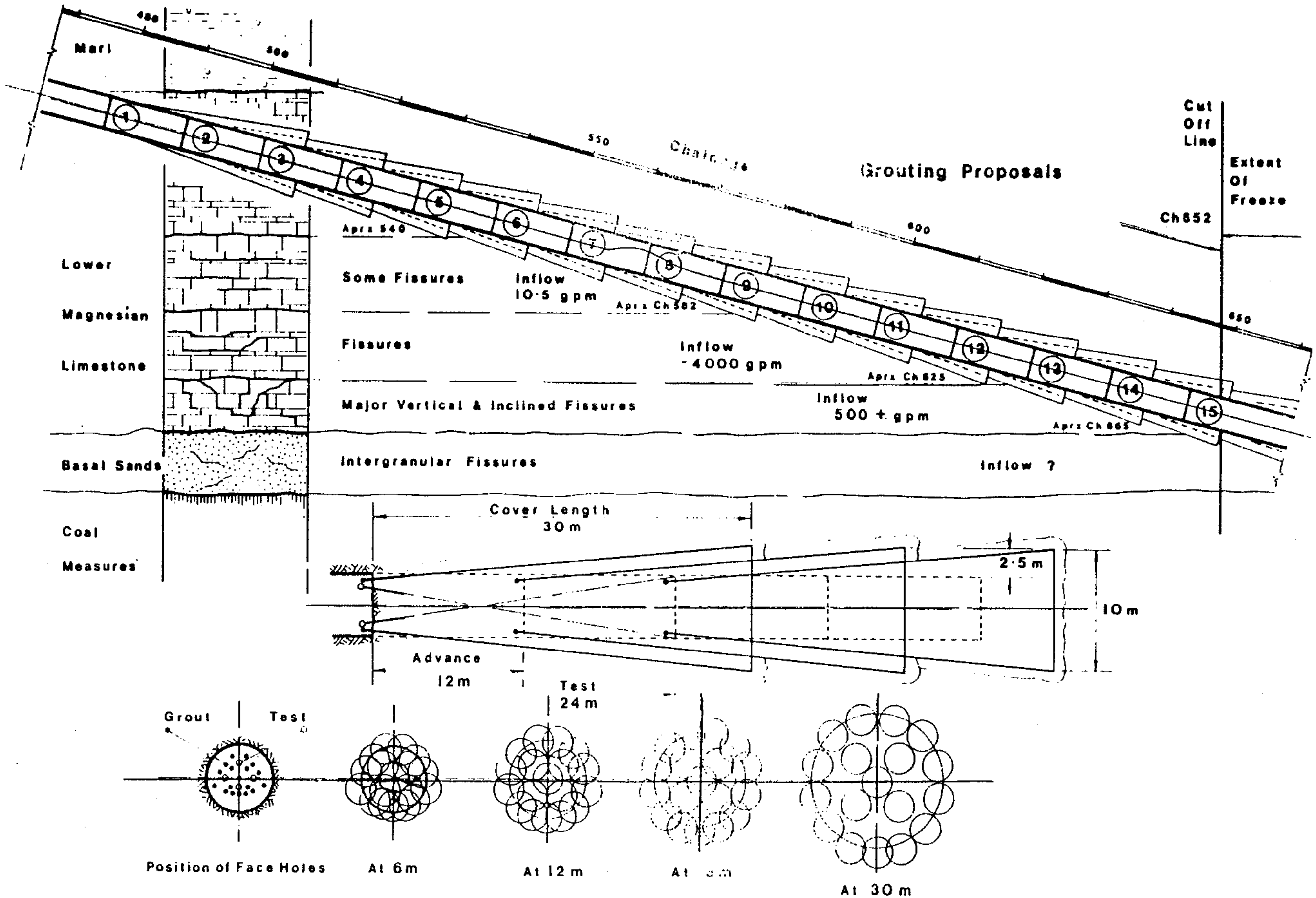


Figure 11 :- Arrangement of Grout Covers & Hole Patterns for Gascoigne Wood Drift No. 1.

TABLE 4: GROUTING OF LOWER MAGNESIAN LIMESTONE AT GASCOIGNE WOOD NO.1 DRIFT

Cover No.	Location of face m	Approx. Depth Range m	Average Inflow/Hole l/s	(gpm)	Quantity Cement Grout Injected tonnes
1	473	100 - 130	6.8	(90)	57.2
2	485	110 - 122	12.1	(160)	38.4
3	497	113 - 125	11.4	(150)	18.0
4	509	116 - 128	11.4	(150)	44.5
5	521	118 - 130	0.1	(20)	12.0
6	533	122 - 133	4.5	(60)	20.5
7	545	125 - 136	0.2	(3)	21.5
8	558	127 - 139	1.1	(15)	21.5
9	570	131 - 142	0.04	(0.5)	1.9
10	582	134 - 145	4.2	(55)	21.1
11	594	136.5- 148	0.01	(0.15)	2.9
12	606	139.5- 151	1.8	(24)	30.1
13	618	142 - 154	0.04	(0.58)	14.3
14	621	142.5- 155.5	0.03	(0.42)	1.3

For the first eleven covers the time taken for drilling and grouting procedures ranged from 8 to 18 days per cover. Somewhat longer was taken with the last two covers with the increased number of injection holes and the very careful approach required in the vicinity of the ice wall. Procedures in this area are described in detail elsewhere (Daw 1983). The complete operation spanned the period from January to October 1979, with a comparable programme carried out almost simultaneously in the No.2 drift. Total residual inflow to the drift through this zone was reduced to less than 4 l/s (52 gpm) by the grouting operation.

Shaft Plug (Temporary Consolidation) - North Selby Mine (Auld 1983)

In December 1982, the No.1 shaft at British Coal's North Selby Mine had reached a depth of 540.2m, with the sump located in the upper section of sandstone of the Ackworth Rock. Previously cover grouting had been carried out from sump levels of approximately 472m, in the Lower Magnesian Limestone, at 507m and 525m in the Brierly Rock. Below the sump lay a major aquifer zone in the middle section of the Ackworth Rock from some 545 to 570m, for which further cover grouting was planned.

During the previous grouting, problems were experienced with the installation of the grout standpipes, due to the poor rock conditions, and deterioration and heave of the sump had occurred. In addition, the long cover lengths of 40m made it difficult to obtain a full seal at the lower levels of injection. For the Ackworth Rock grout cover, therefore, it was decided to take the sump closer than normal to the aquifer zone, and to install a concrete plug. By casting the grout standpipes into the plug, a pressure pad was provided for the next grout cover.

Due to the high potential water inflows from the Ackworth Rock below the plug it was necessary to install a pump lodge for stage pumping to the surface, and the only choice of position was immediately below the last cast section of the shaft wall. (See Fig.12).

At the time of placing the plug, shaft water inflow was approximately 11 l/s (145 gpm). The framework for supporting the grout pipes and water control rising mains during casting of the plug is also shown in Fig.12. A cement replacement material was incorporated in the mix design to reduce the heat of hydration, and additional heat removal was obtained through the rising mains and grout pipes. The design was also such that pressurising for water stopping was possible at the earliest opportunity.

Grouting the actual plug started from the bottom through 50mm grout pipes installed in the rising mains. These pipes were grouted in, leaving the bottom free for injection into the gravel bed, and also secured by high pressure flanges bolted together at the top of the rising mains. The bottom injection was phased to follow backwall injection of the shaft wall above the plug and controlled by using the standpipes as "tell-tales" before closing off for final pressurising.

The shaft water make was reduced to approximately 0.45 l/s (6 gpm) before final tightening up, this quantity being predominantly from behind the shaft lined above the pump lodge. To enable the plug to be subsequently broken out without damaging the shaft wall, the bottom surface of the wall was painted with a bond breaking agent, the hanging rod ends were sleeved and two water bars were incorporated, the inner one protected and the outer one sacrificial for plug sealing.

Following securing of the plug, the Ackworth Rock grout cover was carried out successfully and the concrete plug subsequently removed and shaft sinking continued.

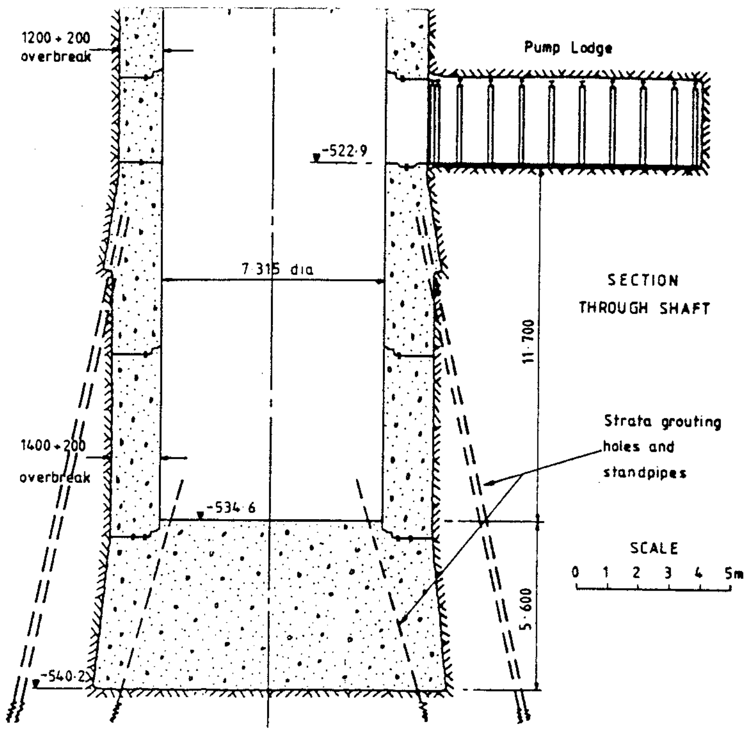
Roadway Dams

British Gypsum Ltd., Sherburn Mine - Emergency Plug (Auld 1983)

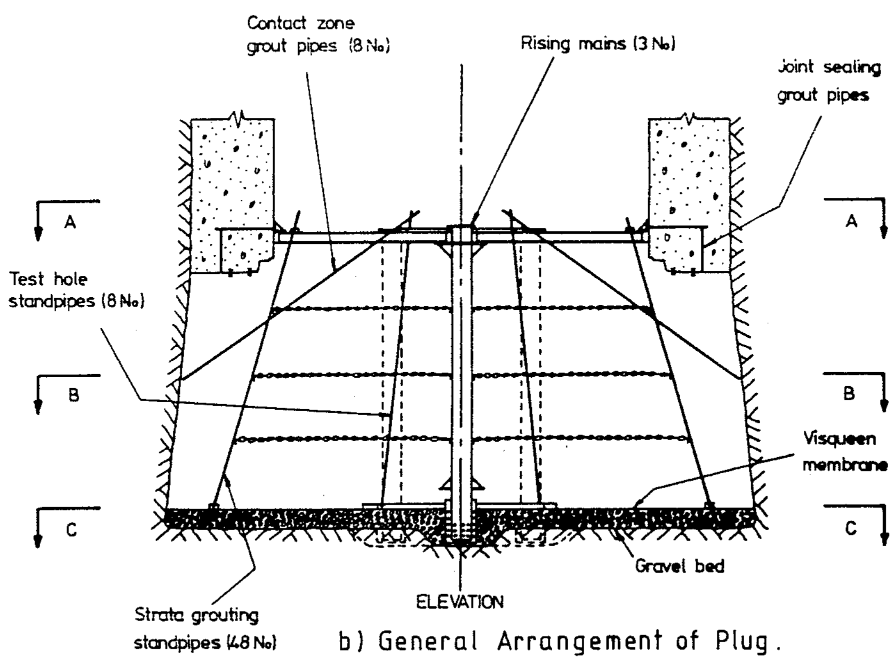
In 1980 British Gypsum constructed a pressure pad in an attempt to seal off water inflow which had developed into the area of the pump sump. The main access to the mine was via a 1 in 4 adit which was close to the inflow position. When the pressure pad failed during grouting operations the inflow was estimated to be 182 l/s (2400 gpm). Cementation Mining Ltd. were asked to design a new scheme for sealing off the water, which continued to rise to some 379 l/s (5000 gpm) and was threatening the mine. The complete plug scheme adopted is shown in Figure 13.

Another gravel bed was laid over the top of the remaining sections of the original pressure pad, containing six additional water control pipes. These pipes carried the water to a new sump position adjacent to the proposed plug site, and additional rising mains were installed in the shaft to cope with the increasing inflow. The various stages of concreting are also shown in Figure 13, and because of the large mass of concrete it was necessary to incorporate construction joints and a cement replacement material in the mix design.

The concrete was pumped from the surface down the 1 in 4 adit, through a 100mm pipe directly into position in the plug. Minimal true design was

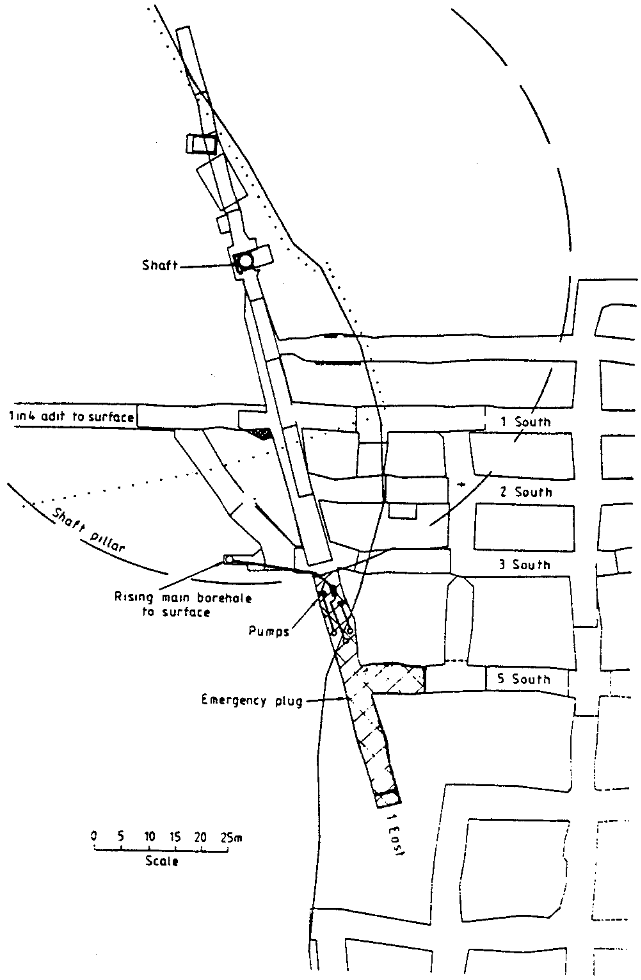


a) Section Through Shaft.

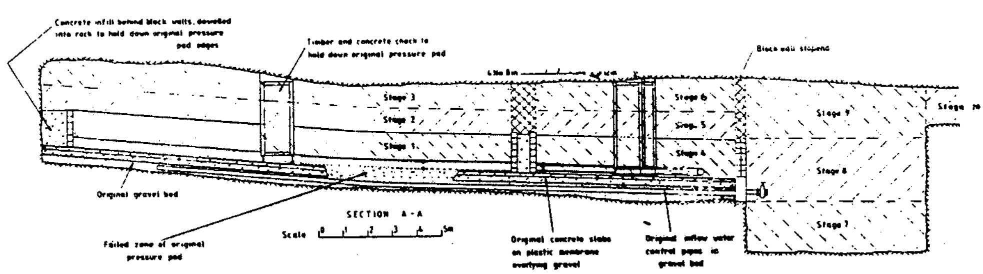


b) General Arrangement of Plug.

Figure 12 :- Temporary Consolidation Plug at North Selby No.1 Shaft.(9)



a) Underground Layout Showing Location of Plug.



b) Longitudinal Section Through Plug Showing Sequence of Concreting

Figure 13 :- Emergency Plug at British Gypsum Ltd. Sherburn Mine.(9)

required for the plug as the depth below ground was only 48m resulting in a hydrostatic pressure of 470 kPa (68 psi), and the plug was extremely long - the latter dictated by practical rather than purely technical considerations. The four week period for placing the concrete resulted from various equipment, labour, and general construction problems, but once concreting had commenced the water inflow was controlled at a peak level of 606 l/s (8000 gpm).

On completion of the various stages of concreting the control pipe valves were closed and the inflow was stopped almost completely. Final sealing by cement grout injection involved a combination of grout pipe positions. Some of these were previously cast into the plug to reach positions not otherwise accessible by drilling from the plug faces. Additional injections through the water control pipe and into the contact zones through holes drilled in the plug faces, enabled the water to be sealed off on a permanent basis.

Bolsover Colliery Underground Dams (Auld 1986)

In 1984 five underground dams were placed by Cementation Mining Ltd. for British Coal (then N.C.B.) in order to protect the workings at Bolsover Colliery from a sudden inrush or overflow situation from the adjacent Arkwright Colliery following its closure and the switching off of its pumps.

For this work Cementation used technology developed at their Gascoigne Wood contract for transporting a structural cementitious mix through a small diameter pipeline from surface to the required underground location. In this instance the mix was transported 372m vertically and then a maximum of 262m horizontally in a single 1½ in. nominal bore steel pipeline, and discharged directly at the dam sites.

A particular problem at Bolsover was that of shaft inaccessibility, but due to the relatively light weight of the pipeline it was attached to a steel rope suspended from a winch down the full length of the 372m deep, sealed off No.1 shaft (See Fig.14).

With this arrangement it proved possible to place the cementitious mix at rates of 4-5 m^3/hr into the dams furthest inbye and at 5-6 m^3/hr in those closest to the shaft. The largest dam was installed in a 5.2m wide by 4m high D-shaped roadway and was 7.4m in length. It contained 194 m^3 of concrete and was poured in 4 lifts of 4m. At a placing rate of about 5 m^3/hr it was possible to complete each lift in just over a single eight hour shift.

Temperature monitoring was carried out in another of the large dams and indicated a maximum temperature of 70°C, with differential temperatures not exceeding 22°C, from an initial temperature of 18°C. No visible shrinkage or thermal cracks were detected, and only small amounts of cement grout were injected in the final tightening phase.

Concrete strengths achieved in the dams were 30 N/mm^2 at 28 days and 70 N/mm^2 at one year. Such high quality, high strength mixes which are capable of being transported long distances underground, are only possible by the incorporation of cement replacement materials and admixtures.

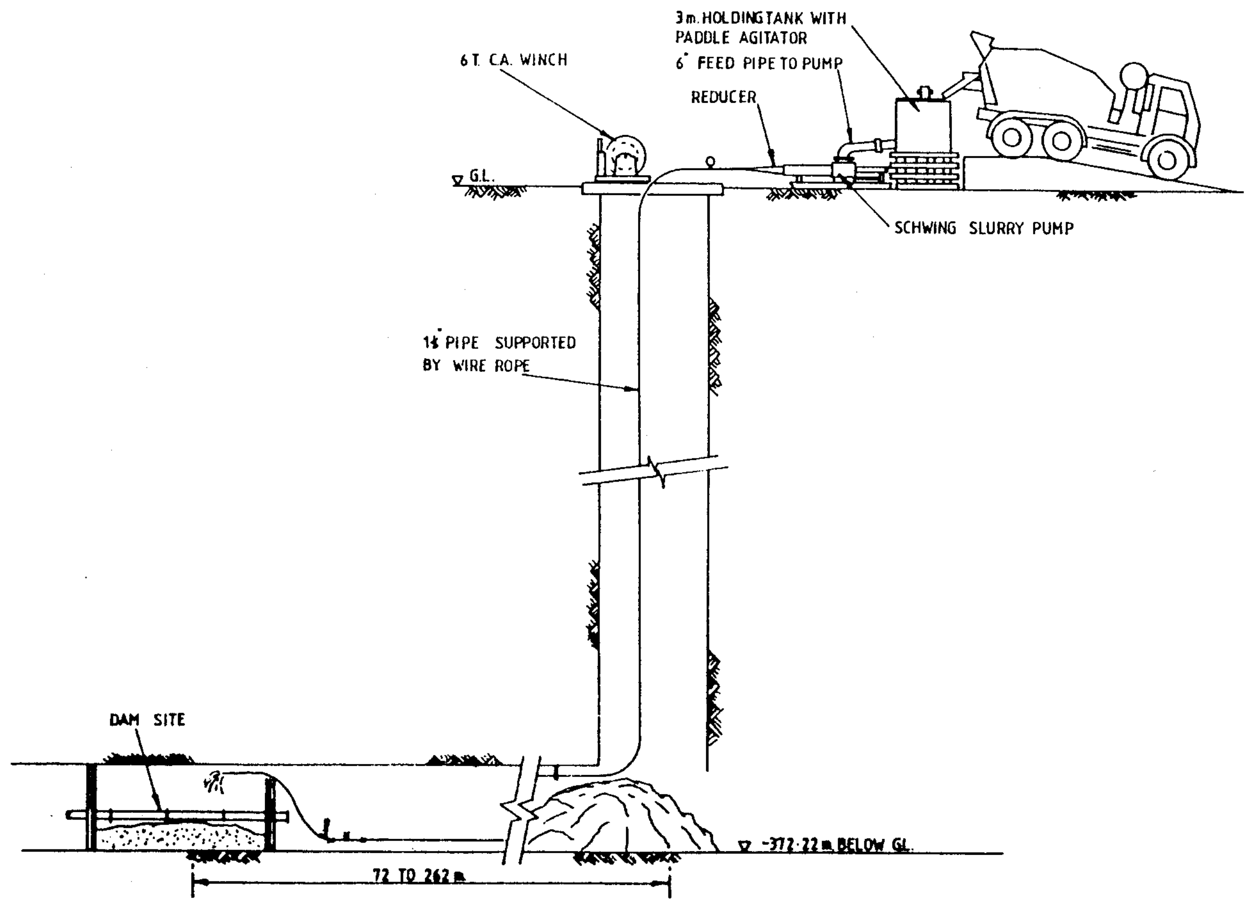


Figure 14:- Schematic Arrangement for Placing Concrete Dams at Bolsover Colliery.(23)

Remedial Work to Shafts

Belle Isle Salt Mine, Louisiana U.S.A. (Greenwood 1982)

This particular example, although some time ago, is of interest as it involves both the recovery of a shaft and the use of the squeeze grouting method.

Fig.15 shows the interconnection of the two shafts with roadways on two different levels separated by about 17m. An accident occurred in 1973 resulting in an inrush mainly of silty sand down the No.2 ventilation shaft. Some 30,000 m^3 of alluvium "flowed" for some 300m from the shaft bottom at a speed of about 1 m/s and filled three roads and crosscuts at the 370m level. It is likely that liquefaction occurred temporarily during flow which accounts for the extensive travel and poor and irregular compaction at rest.

A concrete plug had to be constructed in order to recover the shaft safely, and to facilitate this the inrush material was consolidated by squeeze grouting (Fig.15). First of all a void at the roof where the filling had settled was grouted with a sand/cement mix to allow squeezing in later injections. Squeeze grouting was then started in 0.9m descending stages with injections limited to 0.43 tons of cement and a pressure of 4.86 N/mm^2. An average of eight repeat injections were required before it proved possible to drill further without collapse. Some interconnection occurred between holes drilled on a 1.8m square grid from the overlying air entry at the 354.3m level. Initially the upper parts of the fill in the 7m square roadway were moist and loose, with the lower 2m saturated and "soupy". After consolidation by the squeeze grouting it proved possible to excavate the material and to construct the plug.

Once the shaft had been secured the concrete plug and consolidated fill were excavated completely. The grouting had been so successful and provided such good contact with the roof and walls over the last 12m-15m of excavation that blasting was necessary to remove it.

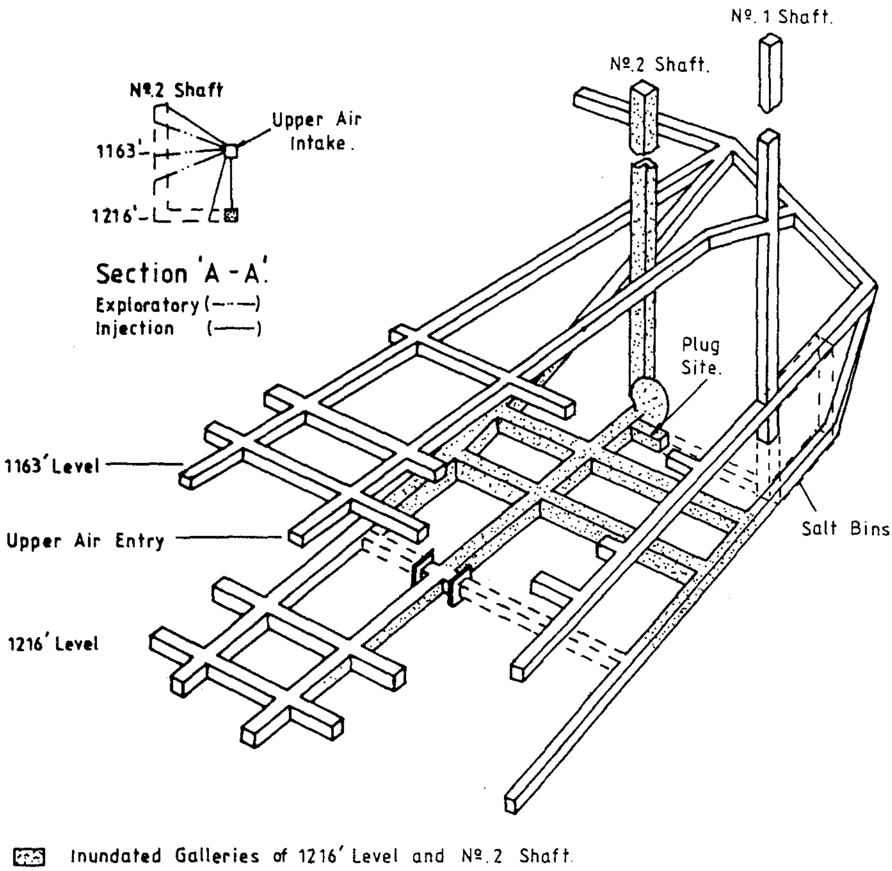
C.S.M.E. Varangeville Emergency Shaft, France

The emergency shaft at Varangeville was originally sunk to a depth of 64.15m and lined with brick to give an internal diameter of 1.6m. The shaft was later lined with steel to 1.4m diameter, deepened to the lower salt bed at 101.5m, and subsequently lined from surface to total depth with a 1.0m diameter steel lining.

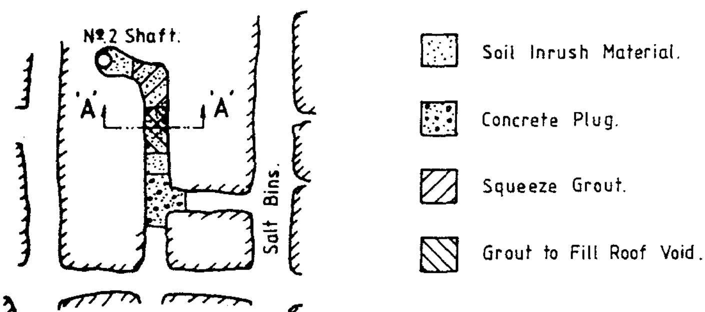
The seepage of brine into the shaft was first observed in 1972, and by 1981 had increased to 0.5 litre/sec (6.6 gpm). The source of brine was the brine stream at the top of the salt bed, which tracked down behind the steel lining to enter the shaft at certain points, notably via a buckled steel liner plate at -66.0m.

It was decided that the shaft would be sealed by infilling with concrete. (See Fig.16).

The shaft furnishings were stripped out from surface to total depth. Temporary garlands were installed at -51.5m and -70.0m, and a relief pipe caulked in the buckled liner plate at -66.0m. A steel support frame was erected in the roadway at the base of the shaft and an initial concrete



Mine Roadways in Vicinity of Shaft Bottom.



Plan of Vicinity of Nº.2 Shaft on 1216' Level After Grouting and Plug Construction.

Figure 15 :- Shaft Recovery by Squeeze Grouting and Plug Constructior at Belle Isle Salt Mine, Louisiana, U.S.A.(8)

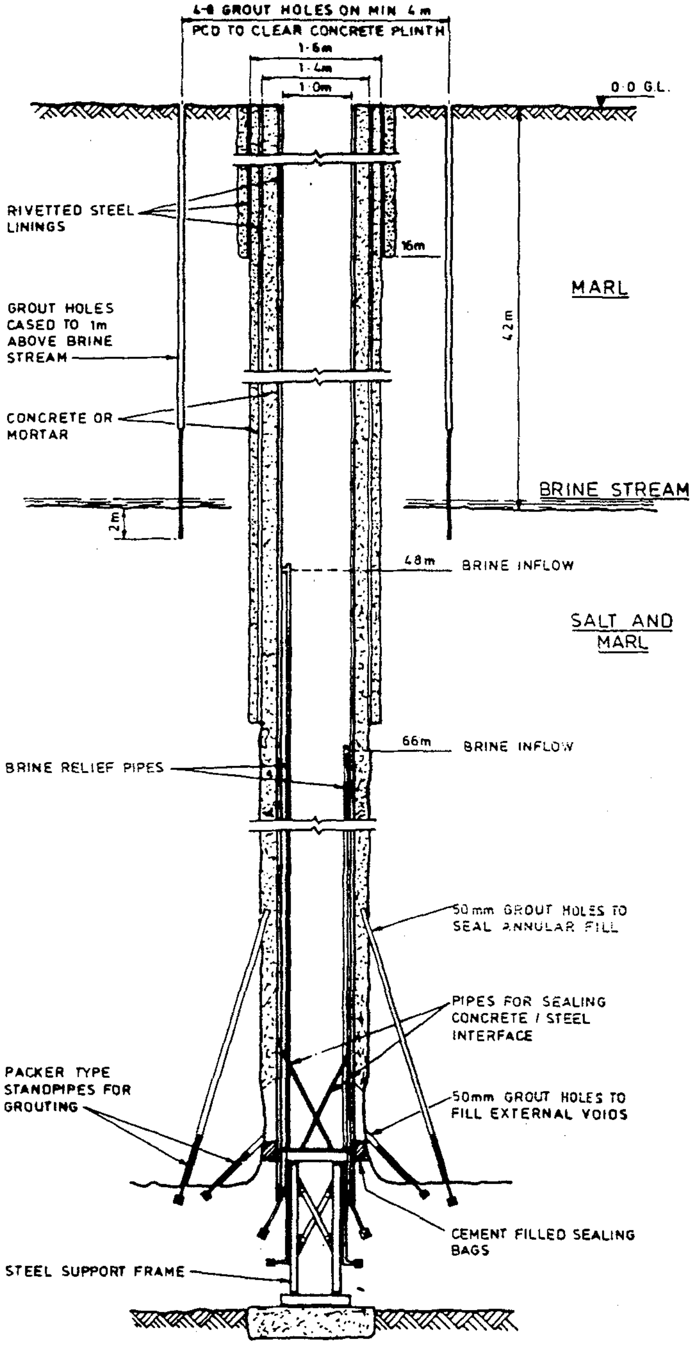


Figure 16 :- Grouting Arrangement for Sealing of Emergency Shaft at Varangeville Mine France.

plug poured to shaft depth -94.5m. This was allowed to cure for 10 days, and the infilling completed with a further three pours. The bottom of the infilled shaft was then sealed by injecting brine/cement grout into a series of holes which had been drilled to intersect contact zones and voids. A summary of the grout injected, in chronological order, is given below in Table 5:

TABLE 5: GROUTING DETAILS FOR SEALING SHAFT AT VARANGEVILLE MINE

Hole	Brine/Cement	Connections	Quantity kg
13	0.9	12	750
12	0.9	15,5,11,7	1800
11	0.9	5,8,1,2	975
5	0.9	8,2	375
8	0.9	4,2,1,9,10,3	300
1	0.9	9	400
4	1.4	-	100
9	0.9	-	125
10	0.9	-	100
6	1.8	-	50
3	1.8	-	225
2	1.8	-	150
Lower holes	1.8	-	1200

The grout holes were then re-drilled and a secondary injection of thin brine/cement grout injected. A total of 400 kg cement was injected into holes 7, 12, 2, 3, 1.

A series of 6 No. holes was drilled from surface down to the brine streams at 43.5m prior to the injection of the main seepage point at -66.0m via the previously installed relief pipe. This injection was commenced with brine cement grout at a mix ratio of 1.4 and decreased in stages to 0.9. A total of 6.3 tonnes of cement were injected.

Subsequent tests carried out on the six surface boreholes found them to be blocked at depths between -36.2m and 43.1m, and only holes 2 and 6 accepted brine from pumping-in tests. Both these holes were then injected with brine/cement grout at mix ratios varying between 1.4 and 0.68 at a total pressure of 1.7 N/mm^2 (246 psi) at the elevation of the brine stream. A total of 2.5 tonnes of cement was injected. The holes were then grouted up to the surface using thick grout injected through 25mm bore pipes which extended to the bottom of each hole.

The shaft sealing was concluded by removing the temporary support frame and shuttering at the bottom of the shaft and excavating a 0.20m wide x 1.8m high annular void. This annulus was shuttered and subsequently filled with thick brine cement grout via 6 vertical steel pipes, using a total of 0.9 tonnes cement.

Although 'bone-dry' when completed, some 3 years later a slight weep (approx. 0.2/day) has occurred due to brine tracking down through the severely corroded steel lining.

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